A Modified Three-Strut (MTS) Model for Masonry-Infilled Steel Frames with Openings

A. Mohebkhah[1], A.A. Tasnimi[1], and H.A. Moghadam[2]

1. Department of Civil Engineering, Engineering Faculty, Tarbiat Modares University, Tehran, I.R. Iran, email: amoheb@modares.ac.ir
2. Department of Civil Engineering, Sharif University of Technology, Tehran, I.R. Iran

ABSTRACT: *Nonlinear numerical modeling of masonry-infilled frames is one of the most complicated problems in structural engineering field. This complexity is attributed to the existence of joints as the major source of weakness and material nonlinearities as well as the infill-frame interaction. Although there are many numerical studies on micro-modeling of solid masonry-infilled steel frames, however, few researches have been conducted on infilled frames with openings. This paper develops a two dimensional numerical model using the specialized discrete element software UDEC (2004) for the nonlinear static analysis of masonry-infilled steel frames with openings. This model is employed to investigate the effect of the size of central window openings on the lateral strength and stiffness of infilled steel frames. Furthermore, the efficient three-strut macro-model proposed for pushover analysis of solid infilled frames is modified for those having central window openings. It was found that the modified three-strut (MTS) model can be used confidently to predict both the stiffness and capacity of such frames up to failure. This model can be easily employed in seismic vulnerability analysis of existing steel frames having infill panels with central window openings.*

Keywords: Masonry-infilled steel frame; Discrete element method; Opening; Nonlinear static analysis; Numerical modeling; Strut

1. Introduction

Steel and reinforced concrete framed structures in urban areas are usually infilled with interior and exterior masonry walls. The resulting system is referred to as an infilled frame, which has high in-plane stiffness and strength. At low levels of lateral forces, frame and infill wall act in a fully composite fashion. However, as the lateral forces level increases, the frame attempts to deform in a flexural mode while the infill tends to deform in a shear mode. Interaction between frame and infill panel significantly increases the infilled frame lateral stiffness and drastically alters the expected dynamic response of the structure. However, the effect of masonry-infill panels is often neglected in the analysis of infilled frames by structural engineers, as it is in current practice. Such an assumption may lead to substantial inaccuracy in predicting the lateral stiffness, strength, and ductility of the frame.

Since 1950's, extensive studies have been performed on lateral load behavior of masonry-infilled frames both experimentally and analytically. Stafford Smith [35-36], Riddington and Stafford-Smith [29], Liauw and Kwan [16] and Moghadam et al [22-23] have conducted experimental and analytical investigations on the lateral stiffness and strength of steel frames infilled with masonry panels. Comprehensive description of the studies performed until 1987 has been reported in the state-of-the-art report on infilled frames by Moghadam and Dowling [21]. Dawe and Seah [7], Mosalam et al [26] and Flanagan and Bennett [11] have studied the behavior of masonry-infilled steel frames under lateral in-plane loads.

Numerical modeling strategies of infilled frames are divided into two distinct categories, i.e. micro-modeling and macro-modeling. For micro-modeling of masonry-infilled frames, both the surrounding frame and the infill wall component details are established using a numerical method such as finite element method (*FEM*) or discrete element method (*DEM*). In this method, the interaction between masonry blocks along the joints as well as the frame-infill interaction is taken into account. In the literature, Achyutha et al [1] developed a two dimensional elastic finite element model to investigate the effect of size of opening and types of stiffeners on the lateral stiffness of infilled frames with openings. Mehrabi and Shing [20] have proposed a smeared-crack nonlinear finite element model to study the nonlinear behavior of infilled reinforced concrete frames. This modeling strategy has been also investigated by Mosalam et al [28], Dawe et al [8], Ghosh and Amde [12] and Asteris [2].

Recently, Mohebkhah et al [25] developed a two-dimensional discrete element model to study the nonlinear static behavior of masonry infilled steel frames. In spite of its high accuracy and precision, this methodology can not be used for practical purposes such as the analysis of multistory, multibay framed structures in design offices. This method is only applicable to research purposes.

In order to analyze the behavior of actual masonry-infilled frames, another methodology, i.e. macro-modeling strategy is usually addressed. In this method, the masonry infill wall is replaced by an equivalent system requiring less computational time and effort. The simplest model in this category was proposed by Stafford-Smith [36] and then adopted by Mainstone [19]. According to this model, an equivalent pin-jointed diagonal strut is substituted for the infill panel. The equivalent width of the strut depends on the relative infill-frame stiffness. In the literature since then, a number of macro-models have been proposed by other researchers. Among them, the models proposed by Chrysostomou et al [5], Mosalam [26-27], Saneinejad and Hobbs [31], and El-Dakhakhni et al [9] are distinguished. In the model suggested by Chrysostomou et al [5], the infill is idealized with three compression-only inclined struts in each direction, which follow the behavior defined by the strength envelope and hysteretic loop equations. The off-diagonal struts are located to represent the interaction between the infill and confining steel frame. Mosalam et al [27] proposed a macro-model in which the infill panel is represented by an equivalent nonlinear truss with contact and tie (tension) elements. The disadvantage of these two models is that, the properties of each model must be chosen in such a way to match the experimental or numerical findings. In other words, some experimental tests or sophisticated numerical analyses have to be done prior to the use of these macro-models.

Saneinejad and Hobbs [31] developed an inelastic analysis and design method for infilled steel frames subjected to in-plane forces which later was adopted by Madan et al [18] and implemented in software IDARC for dynamic analysis of such frames. El-Dakhakhni et al [9] adopting the analysis methodology and concept of Saneinejad and Hobbs' model [31], proposed a simple nonlinear macro-model to estimate the stiffness and the lateral load capacity of masonry-infilled steel frames failing in corner crushing mode. In this method, each masonry panel is replaced by three struts (one diagonal and two off-diagonal) with nonlinear force-deformation characteristics. The advantage of the method is that, it can be easily computerized and implemented in nonlinear analysis of actual multistory, multibay masonry-infilled steel frames.

In most cases, door or window openings are provided in masonry infill panels because of functional and ventilation requirements of buildings. Introducing openings in an infill wall alters its behavior and adds complexity and difficulties in analysis. Furthermore, due to the presence of openings in infill panels, the lateral strength and stiffness of infilled frames is reduced. This reduction in strength and stiffness has not been considered in the above-mentioned macro-models. That is, the models are only applicable to the nonlinear analysis of solid masonry-infilled frames. On the other hand, it has been strongly emphasized by *FEMA*356 [10] that the effect of masonry infilled frames with and without openings must be taken into account on the seismic vulnerability analysis of existing framed buildings. However, no straightforward macro-model is provided in this document to include the effect of masonry-infill panels with openings in the analysis of existing frames. According to *FEMA*356, the strength and stiffness of such frames should be based on nonlinear finite element analysis (micro-modeling) of a composite frame substructure with infill panels that account for the presence of openings. Hence, it is an urgent need to develop a reliable macro-model to predict lateral load carrying capacity, stiffness and components' internal forces at ultimate load of infilled frames with openings.

The purpose of this paper is to propose a nonlinear macro-model for the lateral load analysis of masonry-infilled steel frames which accounts for the presence of central window openings. For this purpose, first a two-dimensional discrete element model (micro-modeling strategy) using the specialized discrete element software *UDEC* [13] is developed and validated against the available experimental data in the literature for the nonlinear static analysis of infilled steel frames subjected to in-plane monotonic loadings. Then, the model is used to study the effect of different opening sizes on the lateral stiffness and collapse load of such frames parametrically. Finally, having the results of parametric *DEM* study, the three-strut model proposed by El-Dakhakhni et al [9] is adopted and modified to consider the effect of central window openings on lateral load behavior of these kinds of frames. This model can be easily employed in seismic vulnerability analysis of existing frames having infill panels with window openings.

2. Discrete Element Method

Sharma et al [32] reports that the discrete element methods were initially developed in 1971 by Cundall for the study of jointed and fractured rock masses. In the discrete element method, various behavioral parameters such as large displacements, rotations, sliding between blocks, crack opening, complete detachment of the blocks, and automatic detection of new contacts are allowed while the analysis is in progress. The calculations performed in the discrete element method alternate between the application of a force-displacement law at all contacts and Newton's second law at all blocks or nodes [13]. The force-displacement law is used to find contact forces from known displacements. According to Newton’s second law, the motion of the blocks relevant to the known forces acting on them would be known.

So far various discrete element applications to masonry structures have been reported for both static and dynamic analysis [3, 4, 14, 15, 33, 34]. Mohebkhah and Tasnimi [24] developed a two-dimensional *DEM* model to investigate the seismic behavior of confined and reinforced brick masonry walls. Mohebkhah et al [25] used a two dimensional *DEM* model for the nonlinear static analysis of masonry-infilled steel frames with openings subjected to in-plane monotonic loading. All of the *DEM* analyses in this study are performed using the developed model in Ref. [25] which is briefly explained in the following section.

3. Description and Validation of the Micro-Modeling Technique

In the above-mentioned *DEM* model, the masonry infill panel is modeled at a semi-detailed level (micro-modeling strategy). This implies that the joint is modeled as an interface with zero thickness. In this approach, fictitious expanded block dimensions are used that are of the same size as the original dimensions plus the real joint thickness. It follows that the elastic properties of the expanded block and the interface joint must be adjusted to yield correct results. The interface's stiffness is deduced from the stiffness of the real joint as follows [30]:

$$k_n = \frac{E_b E_m}{h_m (E_b - E_m)} \tag{1}$$

$$k_s = \frac{G_b G_m}{h_m (G_b - G_m)} \tag{2}$$

Where E_b and E_m are Young's modulus, G_b and G_m are shear modulus, respectively, for block and mortar and h_m is the actual thickness of the mortar. The accuracy of this methodology has been verified by Lourenco [17] using some detailed discontinuum finite element analyses.

The inelastic, isotropic model is used for the behavior of the blocks. The blocks are considered fully deformable, thus allowing deformation to occur both in the blocks and joints and a better simulation of crack propagation and sliding in the joints. It has been shown that when the shear displacement increases, the masonry block cohesion gradually decreases to zero. Hence, the concrete blocks are built using a strain-hardening/softening material model. This model is based on the *UDEC* Mohr-Coulomb model with tension cut-off in conjunction with non-associated shear and associated tension flow rules. Since the steel frame components in the model are expected to behave inelastically at ultimate state of loading, a Von-Mises material model is chosen to represent the steel frame behavior. The Von-Mises criterion is not available in *UDEC*. However, the Drucker-Prager criterion can be degenerated into the Von-Mises criterion for $\phi = 0$ [13]. Althougth the steel frame components are made up of steel I-sections, they are modeled as solid blocks of steel with equivalent elastic and inelastic mechanical properties.

The analysis is carried out sequentially. First, each model is brought to equilibrium under its own dead weight. In order to determine the collapse load, it is often better to use displacement-controlled boundary conditions rather than force-controlled. Therefore, an incremental horizontal displacement is applied at the top corner of the models.

The developed discrete element model is employed here to simulate the in-plane behavior of a concrete masonry-infilled steel frames tested at the University of New Brunswick by Dawe et al [8]. Detailed experimental results of the specimens have been summarized in Dawe and Seah [7]. Among the 28 large-scale specimens tested under racking load in their program, specimen *WC*3 is chosen. The specimen is a single panel 3600 mm long by 2800*mm* high concrete masonry-infilled steel frame with $0.8 \times 2.2m$ central door opening. Infill panel consisted of $200 \times 200 \times 400mm$ concrete blocks placed in running bond within a surrounding moment-resisting steel frame fabricated using $W250 \times 58$ columns and a $W200 \times 46$ roof beam. In each case, a horizontal load applied at an upper corner of the frame was gradually incremented up to the failure load level. The average compressive and tensile strength of the concrete masonry blocks are 31 and 1.0*MPa*, respectively [7]. For the joints, simulating the characteristics of the mortar, a Mohr-Coulomb slip model is employed. The average angle of internal friction and cohesion of the mortar joints are 35^o and 0.6*MPa*, respectively [7].

Figure (1) illustrates the load-displacement diagrams from the experimented specimen, as well as the numerical results, up to a deformation at which the failure mechanism is formed. The local peaks in the diagrams are moments at which a new joint crack occurs or plastic behavior takes place in the blocks. As can be seen, the agreement between experimental (285*kN*) and numerical (297*kN*) collapse loads is satisfactory with an error of 4%. The obtained results reveal the suitable capacity of the *DEM* to model masonry infill walls behavior.

4. Effect of Opening Size on the Lateral Stiffness and Strength

To investigate the effect of the size of central window openings on the lateral strength and stiffness of infilled steel frames, a parametric study is conducted using the *DEM* micro-model that was developed in the previous section. The effect of opening size on the lateral capacity of the infilled frames is studied for various values of a parameter denoted by α that was defined as the percentage of elative ratio of the opening area to the solid infill panel area.

To this end, the infilled steel frame analyzed in Sec.3 is considered with the same properties and a central window opening of different sizes. The models which are to be analyzed are *WS*, *WO*100, *WO*120, and *WO*160. The symbols *WS* and *WOx* stand for the reference solid infilled frame and infilled frame with a central window opening of *x* dimensions in centimeters, respectively. The windows are considered to be square in shape.

Figure (2) illustrates the comparison between the numerical load-displacement diagrams of all the above-mentioned models up to a deformation at which the failure mechanism is formed. As can be seen, due to the presence of openings in infill panels, the lateral strength and stiffness of infilled frames is reduced. The results of these discrete element analyses are also shown numerically in Table (1). According to the seventh column of this table, it can be concluded that the secant stiffness of the infilled frames with openings at the peak load is about 0.39 times the initial stiffness. The lateral strength is also expressed is terms of the dimensionless parameter λ_m, defined as the ratio of the infill panel strength with openings to that of without openings.

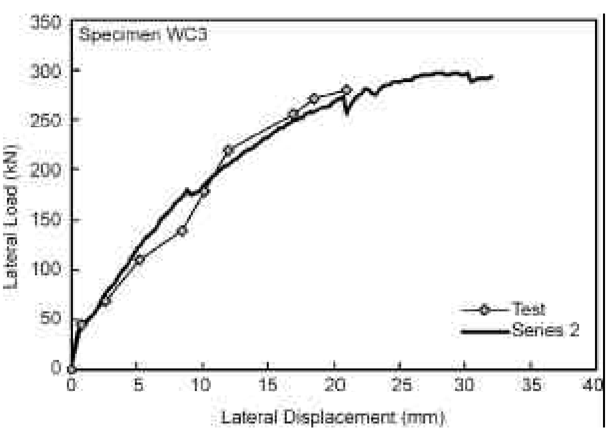

Figure 1. Experimental and numerical lateral load-displacement diagrams for specimen WC3.

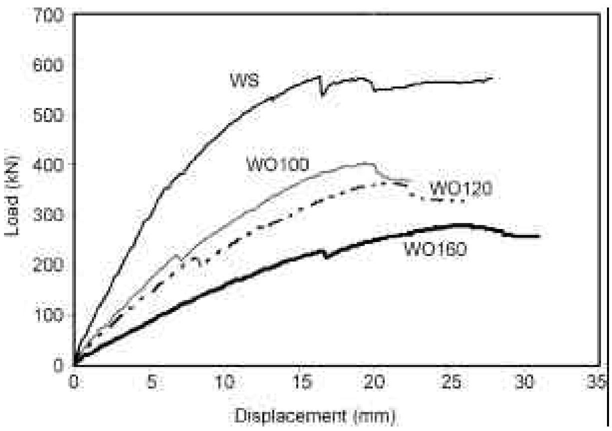

Figure 2. Lateral load-displacement diagrams for all the models using DEM.

Table 1. Discrete element analysis results of the infilled frame models

Model	$\alpha = A_o/A_p$ [%]	H_u (kN)	d_u (kN)	k_i (kN/mm)	$k_s = H_u/d_u$ (kN/mm)	k_s/k_i	λ_m
WS	0	570	16	85.45	35.62	0.42	1
WO100	9.9	400	20	52.27	20.00	0.38	0.668
WO120	14.3	360	22	45.23	16.36	0.36	0.589
WO160	25.4	280	26.5	26.80	10.57	0.39	0.433

Figure (3) demonstrates the *DEM* qualitative results of specimen *WO*120 at a lateral displacement equal to 30*mm* in which as the load increases, cracks occur in the horizontal and vertical interfaces. Then, some stepwise cracks are propagated off-diagonally in the piers. There are some plastic indicators in the program that can be used to assess the state of nonlinear blocks in the numerical model for a static

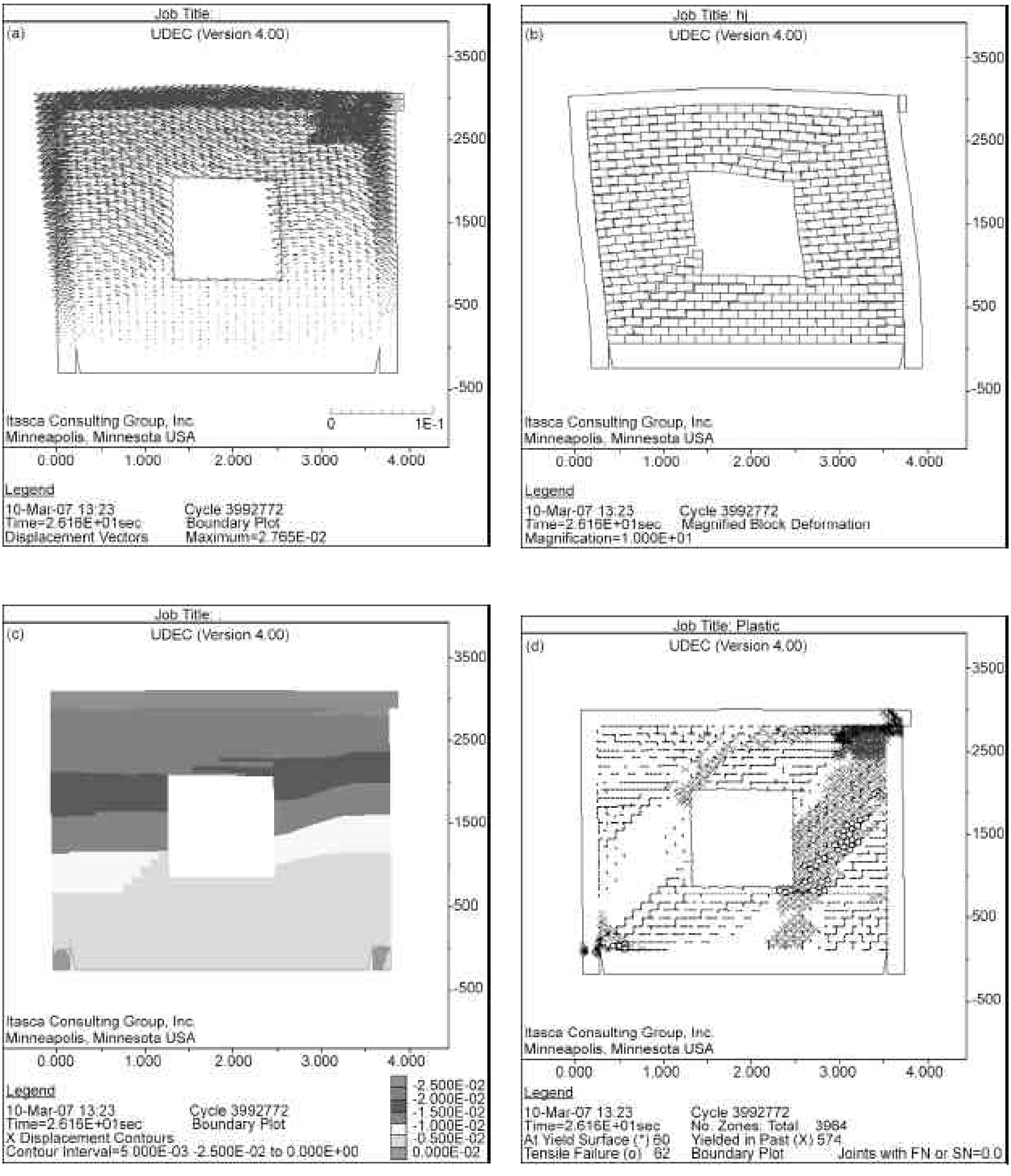

Figure 3. DEM results of model WO120 for a horizontal displacement equal to 27mm: (a) displacement vectors; (b) deformed geometry (magnification factor=10); (c) horizontal displacement contours (m); (d) failure points and crack patterns of the joints; (e) magnified principal stress tensors.

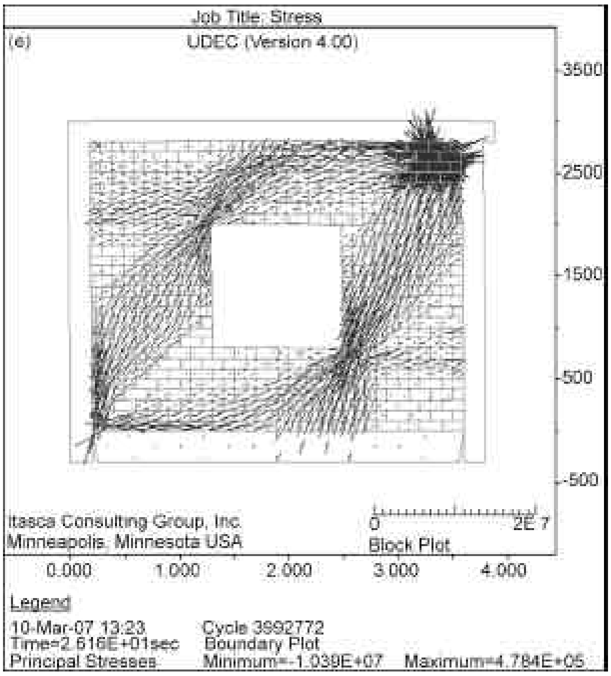

Figure 3. Continued ...

analysis. Such an indication usually denotes that plastic flow is occurring, but it is possible for a block mesh element to sit easily on the yield surface without any significant flow taking place. Figure (3d) shows the collapsed interfaces as well as the plastic behavior in the blocks. The failure situation of the elements is shown in this figure using three distinct symbols: x, * and o indicating "yielding in past", "at yield surface" and "tensile failure", respectively. The situation "yielding in past" indicates the unloaded yielding elements so that their stresses no longer satisfy the Mohr-Coulomb yield criterion. In addition, the situation "at yield surface" illustrates the actively yielding elements which are important to the detection of a failure mechanism. Figure (3e) shows a representation of the principal stress tensors at ultimate capacity. In this figure, the direction of principal stresses shows the activated compressive parts orientation of the infill panel subjected to lateral loading.

5. Description of the Modified Three-Strut (MTS) Model

As was pointed out earlier in this paper, there are a few macro-models to predict the nonlinear behavior of masonry-infilled frames. Among them, the three-strut model proposed by El-Dakhakhni [9], is the most popular and applicable model which can be used for the nonlinear analysis of actual infilled frames failing in corner crushing mode. However, this model is not applicable to the analysis of infilled frames with openings. A simple procedure to overcome this problem is to replace the three-strut system by another equivalent three-strut system. The equivalent system properties can be obtained using the available experimental or numerical results of infilled frames with openings. The disadvantage of this procedure is that, the local response of such frames can not be obtained exactly. To achieve this end, first the model proposed by El-Dakhakhni is briefly defined. Then, owing to the lack of suitable experimental data on the nonlinear behavior of such frames, the numerical results presented in the previous section are employed to modify El-Dakhakhni's model.

Based on the comprehensive analytical and experimental studies of Saneinejad and Hobbs [31], El-Dakhakhni [9] proposed an efficient macro-model for the pushover analysis of the infilled frames up to failure. To develop the model, he assumed that the infill panel is composed of two diagonal regions; one connecting the top beam to the leeward column and the other region connecting the windward column to the lower beam [9]. Hence, the infill panel is replaced by three struts; one diagonal and two off-diagonal, connecting the two loaded corners and the points of maximum moments in the beams and the columns, respectively. The advantage of using this three-strut model rather than the single diagonal strut is that in this model the internal forces (i.e. shear forces and bending moments) in the steel frame members can be estimated. The steel frame members are modeled using elastic beam elements connected by nonlinear rotational spring elements [9]. However, in this paper the source of nonlinearity is assumed to be concentrated in the beams and columns ends (predefined plastic hinges) as it is common in pushover analysis of framed structures. According to Saneinejad and Hobbs [31], using some assumptions and simplifications, the contact lengths (which are approximately the distance from the frame connections to the points of maximum bending moments in the beams and columns) are estimated as follows:

$$\alpha_c h = \sqrt{\frac{2(M_{pj} + 0.2M_{pc})}{tf'_{mx}}} \leq 0.4h \tag{3}$$

$$\alpha_b l = \sqrt{\frac{2(M_{pj} + 0.2M_{pb})}{tf'_{my}}} \leq 0.4l \tag{4}$$

The above lengths determine the location of two off-diagonal struts end connection points to the frame members. Again, the total equivalent diagonal region area proposed by Saneinejad and Hobbs [31], is simplified as [9]:

$$A = \frac{(1-\alpha_c)\alpha_c ht}{\cos\theta} \quad (5)$$

The collapse load of masonry-infilled steel frames is resulted from the contribution of two parts; the resistance of infill panel (R) and steel frame as follows [31]:

$$H = R\cos\theta + \frac{2M_{pj}}{h} \quad (6)$$

$$R = \frac{hH - 2M_{pj}}{h\cos\theta}, \qquad A = \frac{R}{f'_m} \quad (7)$$

A similar relation can be written for an infill panel with opening as follows:

$$R_o = \frac{hH_o - 2M_{pj}}{h\cos\theta}, \qquad A_m = \frac{R_o}{f'_m} \quad (8)$$

Hence, the dimensionless parameter λ_m, to account for the effect of opening size on the lateral load capacity and stiffness of infill panels, can be defined as follows:

$$\lambda_m = \frac{R_o}{R} = \frac{A_m}{A} = \frac{hH_o - 2M_{pj}}{hH - 2M_{pj}} \quad (9)$$

In order to determine the modification factor, the variation of this factor with the parameter α (the ratio of central window opening to infill panel area) is computed in the last column of Table (1). The resulting values of this factor are also plotted in Figure (4) along with the resulting regression equation. As it can be seen, an exponential regression equation ($r^2 = 0.99$) shows a good approximation for the modification factor computed using discrete element method. Therefore, the modified equivalent diagonal region area in infilled frames with a central window opening is given by:

$$A_m = \lambda_m A_d \quad (10)$$

in which

$$\lambda_m = 0.963\, e^{-0.031\alpha}$$

Having the value of A_m and assuming uniform contact stress distribution along the contact areas, the infill panel with opening is replaced by two off-diagonal struts ($S1$), each of area $A_1 = A_m/4$ and one diagonal ($S2$) of area $A_2 = A/2$ [9] as shown in Figure (5).

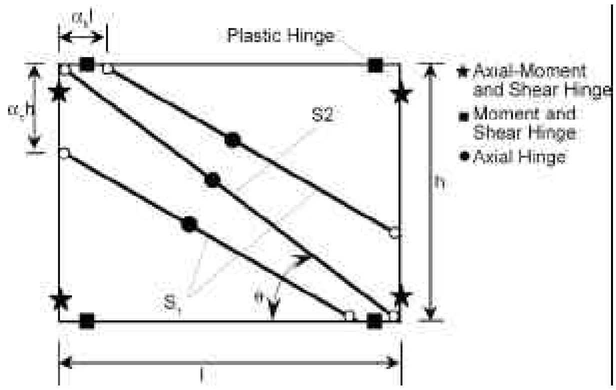

Figure 5. Three struts geometry and plastic hinges placement.

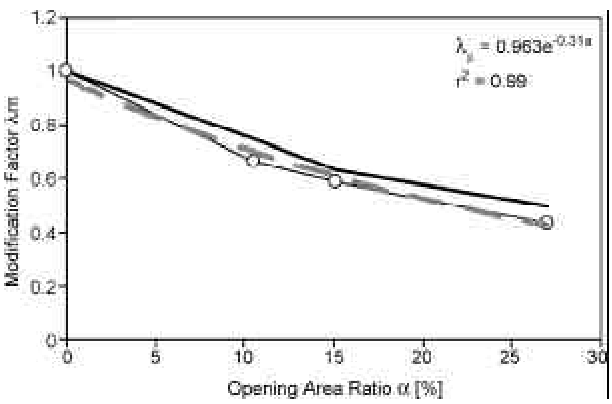

Figure 4. Variation of the modification factor versus the ratio of opening area.

As was pointed out earlier, the source of frame nonlinearity is assumed to be concentrated in the beams and columns ends. To this end, different kinds of plastic hinges are introduced in the frame members as shown in Figure (5) to account for the effect of material nonlinearity. The mechanical characteristics of these plastic hinges can be found in *FEMA*356 [10]. Plastic hinges in columns should capture the interaction between axial load and moment capacity. These hinges should be placed close to the beam face. Hinges in beams should represent the flexural behavior of the members. Shear hinges must also be incorporated in both columns and beams. The three struts, however, only need hinges that represent the axial load. These hinges should be placed at the midspan of the struts. To define the nonlinear behavior of masonry struts' hinges, the simplified trilinear relation proposed by El-Dakhakhni [9] is adopted.

6. Modeling the Infilled Frames Using the Modified Three-Strut Model (MTS)

The modified three-strut model developed in the previous section is used here to simulate the

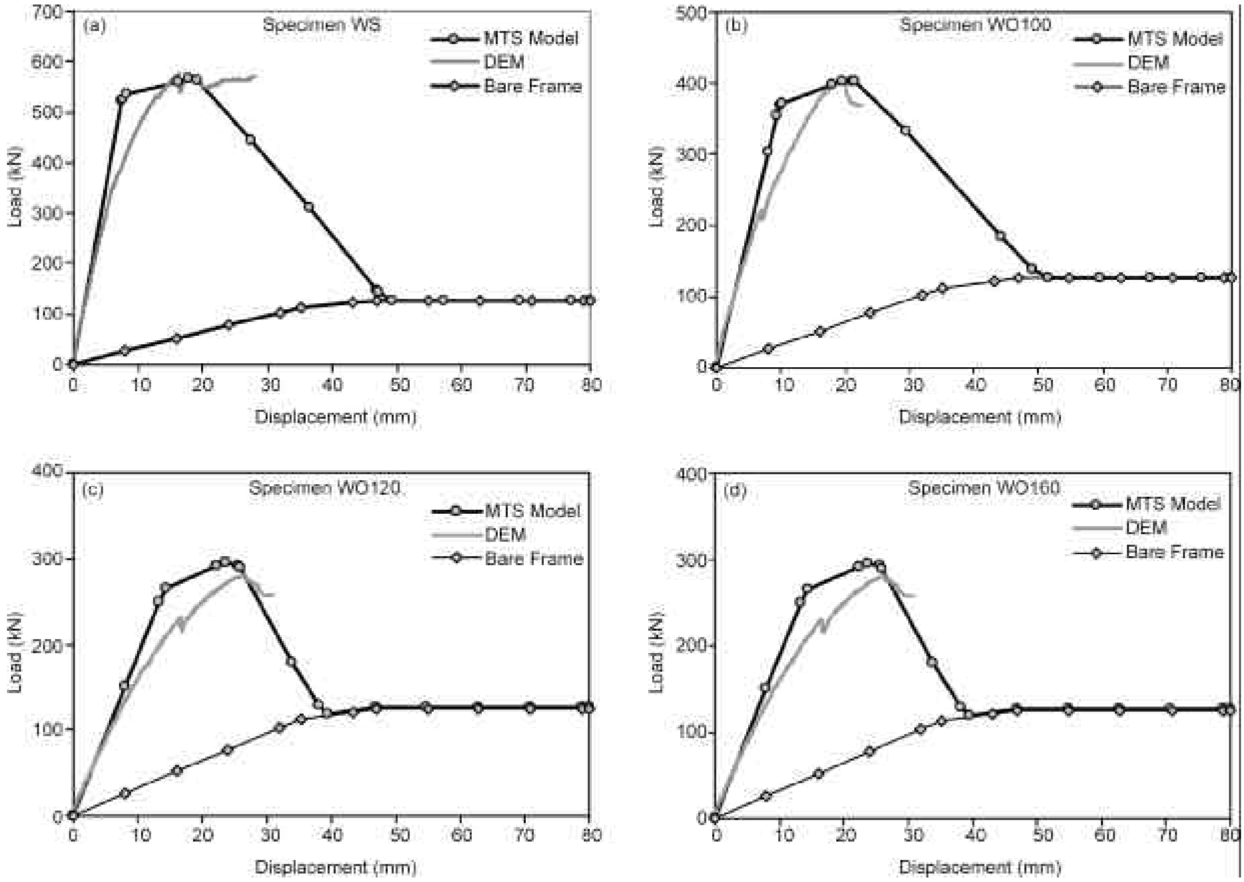

Figure 6. Load-displacement relations for specimen: (a) WS; (b) WO100; (c) WO120; (d) WO160.

nonlinear static behavior of masonry-infilled frames *WS*, *WO*100, *WO*120 and *WO*160. In this regard, a pushover analysis was used using Nonlinear *SAP*2000 program [6] to generate the load-displacement relations of the specimens. The load-displacement relations for the bare and the infilled frames are shown in Figure (6) along with discrete element results for comparison. As it can be seen, the *MTS* model can predict both stiffness and ultimate load capacity of the masonry-infilled frames with openings up to failure.

The maximum error between *DEM* and *MTS* model ultimate loads is about 5%. After damage and degradation of the infill, the load-displacement of the infilled frames reaches the ultimate capacity of the bare frame.

7. Conclusion

In this paper, two kinds of numerical modeling strategies were addressed to simulate the in-plane nonlinear static behavior of infilled frames with openings, i.e. micro-modeling and macro-modeling. For micro-modeling, a two-dimensional discrete element model was developed for the inelastic nonlinear analysis of masonry-infilled steel. The model was validated and used to investigate the effect of the size of central window openings on the lateral strength and stiffness of infilled steel frames. It was found that, the numerical model is applicable to a detailed simulation of the response of such frames throughout the loading process leading to failure. Furthermore, due to the fact that the *DEM* micro-model is not applicable to actual frames, a three-strut macro-model given in the literature was adopted and modified to investigate the nonlinear global behavior of infilled steel frames with central openings. This model is applicable to actual multi-story, multibay masonry-infilled steel frames analysis. However, the proposed model should be more refined to take into account the different opening locations in the infill panel using equivalent truss type models. Also, the model shall be capable of predicting the ductility of such frames correctly.

Acknowledgment

The authors are grateful to enormous help of Dr. W. W. El-Dakhakhni in providing papers and documents regarding this research.

References

1. Achyutha, H., Jagadish, R., Rao, P.S., and Shakeebur Rahman, S. (1986). "Finite Element Simulation of the Elastic Behavior of Infilled Frames with Openings", C*omputers and Structures*, **23**(5), 685-696.

2. Asteris, P.G. (2003). "Lateral Stiffness of Brick Masonry Infilled Plane Frames", *J. of Struct. Eng.*, **129**(8), 1071-1079.

3. Azevedo, J.J., Sincraian, G.E., and Lemos, J.V. (2000). "Seismic Behavior of Blocky Masonry Structures", *Earthquake Spectra*, **16**(2), 337-365.

4. Brookes, C.L. and Mehrkar-Asl, S. (2002). "Numerical Modeling of Reinforced Masonry to Enhance Seismic Resistance", *Proc. of the First Conference on Strengthening and Retrofitting of Structures*, Amir-kabir University, Tehran, Iran.

5. Chrysostomou, C.Z., Gergely, P., and Abel, J.F. (2002). "A Six-Strut Model for Nonlinear Dynamic Analysis of Steel Infilled Frames", *International Journal of Structural Stability and Dynamics*, **2**(3), 335-353.

6. Computers and Structures Inc. SAP2000 (2006). "Three-Dimensional Static and Dynamic Finite Element Analysis and Design of Structures", Berkeley, Educational Version 10.0, Calif.

7. Dawe, J.L. and Seah, C.K. (1989). "Behavior of Masonry Infilled Steel Frames", *Can. J. Civ. Eng.*, **16**, 865-876.

8. Dawe, J.L., Seah, C.K., and Liu, Y. (2001). "A Computer Model for Predicting Infilled Frame Behavior", *Can. J. Civ. Eng.*, **28**, 133-148.

9. El-Dakhakhni W.W., Elgaaly M., and Hamid, A. A. (2003). "Three-Strut Model for Concrete Masonry-Infilled Steel Frames", *J. of Struct. Eng.*, **129**(2), 177-185.

10. Federal Emergency Management Agency (FEMA) (2000). "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", Report No. FEMA 356, FEMA, Washington, D.C.

11. Flanagan, R.D. and Bennett, R.M. (1999). "In-Plane Behavior of Structural Clay Tile Infilled Frames", *J. of Struct. Eng.*, **125**(6), 590-599.

12. Ghosh, A.K. and Amde, A.M. (2002). "Finite Element Analysis of Infilled Frames", *J. of Struct. Eng.*, **128**(7), 881-889.

13. Itasca Consulting Group (2004). "UDEC-Universal Distinct Element Code", Version 4.0-User's Manual", Minneapolis, MN, USA.

14. Karimi, M.S. (2005). "Discrete Element Method Analysis of Masonry Shear Walls", *Proc. of the 2nd International Conference on Concrete and Development*, ICCD, Tehran, Iran.

15. Lemos, J.V. (1985). "Discrete Element Modeling of the Seismic Behavior of Stone Masonry Arches", Computer Methods in Structural Masonry- 4, Eds. Pande, Middleton and Kralj, E and FN SPON, 220-227.

16. Liauw, T.C. and Kwan, K.H. (1985). "Unified Plastic Analysis for Infilled Frames", *J. of Struct. Eng.*, **111**(7), 1427-1448.

17. Lourenco, P.B. (1996). "Computational Strategies for Masonry Structures", Ph.D. Thesis, Delft University of Technology, Delft, The Netherlands.

18. Madan, A., Reinhorn, A.M., Mander, J.B., and Valles, R.E. (1997). "Modeling of Masonry Infill Panels for Structural Analysis", *J. of Struct. Eng.*, 123(10), 1295-1302.

19. Mainstone, R.J. (1971). "On the Stiffness and Strengths of Infilled Frames", *Proceedings of the Institution of Civil Engineers*, Supplement IV, 57-90.

20. Mehrabi, A.B. and Shing, P.B. (1997). "Finite Element Modeling of Masonry-Infilled RC Frames", *J. of Struct. Eng.*, **123**(5), 604-613.

21. Moghadam, H.A. and Dowling, P.J. (1987). "The State-of-the-Art in Infilled Frames", ESEE Res. Rep. No. 87-2, Imperial Coll. of Sci. and Technol., Civ. Engrg. Dept., London, England.

22. Moghadam, H.A. (2004). Lateral Load Behavior of Masonry Infilled Steel Frames with Repair and Retrofit", *J. of Struct. Eng.*, **130**(1), 56-63.

23. Moghadam, H.A., Mohammadi, M.GH., and Ghaemian, M. (2006). "Experimental and Analytical Investigation into Crack Strength Determination of Infilled Steel Frames", *J.*

Construct. Steel Res., **62**(12), 1341-1352.

24. Mohebkhah, A. and Tasnimi, A.A. (2007). "Seismic Behavior of Brick Masonry Walls Recommended by IS-2800: Experimental and Numerical Approaches", *Proc. of the 5th Conf. on Seismology and Earthquake Engineering*, International Institute of Earthquake Engineering and Seismology, Tehran, Iran.

25. Mohebkhah, A., Tasnimi, A.A., and Moghaddam H.A. (2007). "Nonlinear Analysis of Masonry-Infilled Steel Frames Using Discrete Element Method", *Submitted to the J. Construct. Steel Res.*

26. Mosalam, K.M., White, R.N., and Gergely, P. (1997). "Static Response of Infilled Frames Using Quasi-Static Experimentation", *J. of Struct. Eng.*, **123**(11), 1462-1469.

27. Mosalam, K.M., White, R.N., and Ayala, G. (1998). "Response of Infilled Frames Using Pseudo-Dynamic Experimentation", *Earthquake Engng. Struct. Dyn.*, **27**, 589-608.

28. Mosalam, K.M., White, R.N., and Gergely, P. "Computational Strategies for Frames with Infill Walls: Discrete and Smeared Crack Analysis and Seismic Fragility", Technical Report NCEER-97-0021, National Center for Earthquake Engineering Research, State Univ. of N.Y. at Buffalo.

29. Riddington, J.R. and Stafford Smith, B. (1997). "Analysis of Infilled Frames Subjected to Racking with Design Recommendations", *The Structural Engineer*; **55**(6), 263-268.

30. Rots, J.G. Editor (1997). "Structural Masonry: An Experimental/Numerical Basis for Practical Design Rules", *Center for Civil Engineering Research and Codes (CUR)*, Balkema.

31. Saneinejad, A. and Hobbs, B. (1995). "Inelastic Design of Infiled Frames", *J. of Struct. Eng.*, **121**(4), 634-650.

32. Sharma, V.M., Saxena, K.R, and Woods, R.D. Editors (1999). "Distinct Element Modeling in Geomechanics", Rotterdam: A. A. Balkema.

33. Sincraian, G.E. and Azevedo, J.J. (1998). "Numerical Simulation of the Seismic Behavior of Stone and Brick Masonry Structures", *Proc. of the 11th European Conference on Earthquake Engineering*, Balkema.

34. Sincraian, G.E., Lemos, J.V., and Oliveira, C.S. (1998). "Assessment of the Seismic Behavior of Stone Masonry Aqueduct Using the Discrete Element Method", *Proc. 11th European Conf. on Earthquake Engineering*, Balkema.

35. Stafford-Smith, B. (1962). "Lateral Stiffness of Infilled Frames", *J. Struct. Div., ASCE*, **88**(ST6), 183-199.

36. Stafford-Smith, B. (1996). "Behavior of Square Infilled Frames", *J. Struct. Div., ASCE*, **92**(ST1), 381-403.

Nomenclature

A	area of the loaded diagonal region of the infill panel
E_s	Young's modulus of steel
E_b	Young's modulus of concrete masonry block
E_m	Young's modulus of mortar
G_b	shear modulus of concrete masonry block
G_m	shear modulus of mortar
H	lateral load capacity of solid masonry-infilled frame
H_o	lateral load capacity of masonry-infilled frame with opening
M_p	plastic moment capacity of frame members
R	Resistance of solid infill panel
R_o	Resistance of infill panel with opening
α	ratio of central window opening to infill panel area
α_c	ratio of the column contact length to the height of the column
α_b	ratio of the beam contact length to the span of the beam
h_m	thickness of the mortar
h	column height
l	beam span
θ	$\tan^{-1}(h/l)$
k_n	normal joint stiffness $(N/mm^2/mm)$
k_s	shear joint stiffness $(N/mm^2/mm)$
f'_m	compressive strength of the masonry panel
λ_m	modification factor of diagonal region area
t	thickness of the infill panel.